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Analysis of Demand and Leakage Distributing Uniformly Along Pipes

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Abstract

It is well known that leakage is a kind of pressure-dependent flow in water distribution systems. Usually leakages are assumed at linked nodes, with no water flowing out along pipes. Simulation results based on this assumption may create a distortion in the momentum losses, and pressure-dependent leakage may deviate from real value. This paper proposes a new methodology based on the assumption that demand and pressure-dependent leakage distribute uniformly along pipes. The new methodology includes the hydraulic resistance correction model with regard to energy conservation. The results from the traditional and new assumptions are compared and shown that the proposed new method is more realistic, especially in modelling leakage, which is sensitive to pressure.

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Keywords: demand; leakage; pressure-dependent; uniform pipe flow

1. Introduction

Water distribution systems (WDSs) are composed of nodes and links, and the nodes include junction, tank, reservoir, while the links consist of pipes, valves, pumps. With the popularization of information technology and geographical information system, water industry can get all the information in water topology. Although the full scale model of WDSs is available, the accuracy of any computer model is affected by many uncertainty factors not just the model scale [1], moreover, the size of simulation will increase drastically. Therefore model simplification is

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a more practical manner. Traditionally the water usages of house connections are assumed to distribute uniformly along the pipes, and background leakages have the same assumption [2]. For simplification they are lumped into the nearest nodes, and the simplest method is to allocate the pipe discharge equally to both of its nodes. But where topology simplification and leakage as a “pseudo demand” are concerned, this is not fully correct and will creates a distortion in the momentum losses [3], which results in false simulation of pressure-dependent leakage [4]. In the past few years, many researchers have paid attention to this problem, and several approaches have been proposed [5](Todini 2008). At present, the most feasible method is hydraulic resistance correction [6-8]. Berardi et al. [9] considered flow region when hydraulic resistance is corrected, and Giustolisi and Laucelli [10] advanced the above method to simulate both demand and leakage as pressure-dependent parameters.

Due to the sensitivity of leakage to pressure, if demand and leakage which are distributed along pipes are simply allocated to nodes without considering the real distribution characteristic, there will be an adverse impact on the accuracy of the hydraulic calculation, and the calculated leakage will be much different from the actual value. In this paper, based on the method proposed by Franchini and Alvisi [8], EPANET [11] is used to perform hydraulic resistance correction in order to make further investigation on pressure-dependent leakage along pipes.

2. Hydraulic resistance correction model

In actual situation, there are different number of house connections to discharge water from pipes. Although by mean of information technology and geographical information system, the full scale model of WDS is not out of the question, the size of WDS model increases largely, which decreases computational efficiency. And excessive detail may have no use for many WDSs analysis. Therefore more practical way is the assumption that water discharge is uniformly along pipe, as can be shown in Fig. 1 (a) which implies water flows form node A to B.

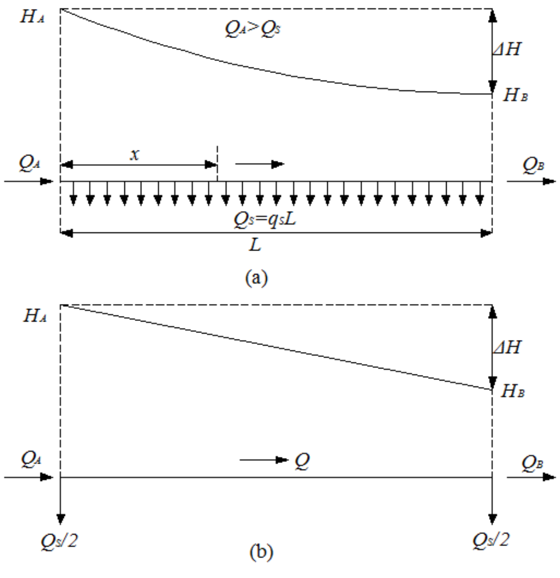

Fig.1. (a) Schematic representation of pipe with uniformly demand and leakage fed from only one node; (b) equivalent scheme of pipe head loss with pipe flow Q.

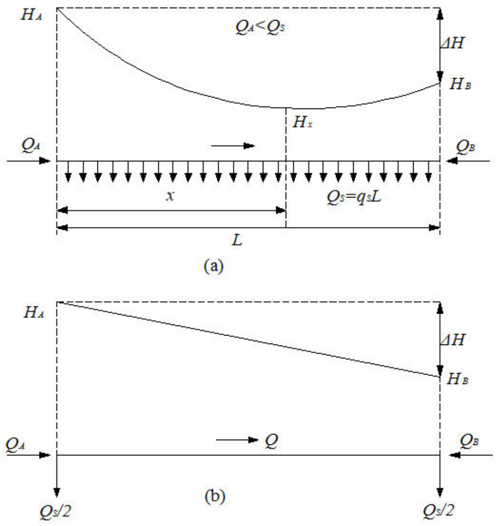

Fig.2. (a) Schematic representation of pipe with uniformly demand and leakage fed from both nodes; (b) equivalent scheme of pipe head loss with pipe flow Q.

Let q_d denote the demand per unit of length, q_l indicate the leakage per unit of length, and as sum of the two above-mentioned, q_s represents the discharge per unit of length. The discharge at point x in Fig.1 is

$$Q_x = Q_A - q_s x = Q_B + q_s\left(L - x\right) \tag{1}$$

The head loss per unit of length is

$$J_x = -\frac{dH}{dx} = aQ_x^n \tag{2}$$

And the total head loss along pipe is

$$\Delta H = \int_0^L J_x dx = \int_0^L aQ_x^n dx = \int_0^L a\left[Q_B + q_s\left(L-x\right)\right]^n dx = \int_0^L a\left[Q_B + \frac{Q_S}{L}\left(L-x\right)\right]^n dx \tag{3}$$

Let $\gamma = Q_B/Q_S$, and then,

$$\Delta H = \frac{aQ_S^n L}{n+1}\left[\left(\gamma+1\right)^{n+1} - \gamma^{n+1}\right] \tag{4}$$

If $Q_A = Q_B + Q_S$ is substituted into the above Equation (4), the total head loss is

$$\Delta H = \frac{aL}{Q_s\left(n+1\right)}\left(Q_A^{n+1} - Q_B^{n+1}\right) \tag{5}$$

When the pipe discharge is allocated equally to nodes as shown in Fig.1 (b), $Q_A = Q + Q_S/2$, the total head loss for this traditional assumption is as follows

$$\Delta H' = aLQ^n = aL\left(Q_B + \frac{Q_S}{2}\right)^n = aQ_S^n L\left(\gamma + \frac{1}{2}\right)^n \tag{6}$$

It is found by comparing equations (4) and (6) that there is head loss error in traditional assumption, and when frictional head loss is evaluated using the Darcy-Weisbach friction formula, the head loss in equation (6) is underestimated by one quarter [3]. Therefore hydraulic resistance correction coefficient ε is introduced to compensate the error.

$$aLQ^n \varepsilon = \frac{aL}{Q_s(n+1)}\left(Q_A^{n+1} - Q_B^{n+1}\right) \tag{7}$$

ε is equal to

$$\varepsilon = \frac{1}{(n+1)Q_s Q^n}\left[\left(Q + \frac{Q_S}{2}\right)^{n+1} - \left(Q - \frac{Q_S}{2}\right)^{n+1}\right] \tag{8}$$

which becomes as follows for n=2

$$\varepsilon = 1 + \frac{1}{12}\left(\frac{Q_S}{Q}\right)^2 \tag{9}$$

and the relation between Q_S and Q can also be derived as show below

$$0 < Q_S/Q < 2 \tag{10}$$

Fig.2 (a) has the same head loss to Fig.1 (a), and in Fig.2 (a) $Q_A = Q_S/2 + Q$, $Q_B = Q_S/2 - Q$, so ε is

$$\varepsilon = \frac{1}{(n+1)Q_s Q^n}\left[\left(\frac{Q_S}{2} + Q\right)^{n+1} - \left(\frac{Q_S}{2} - Q\right)^{n+1}\right] \tag{11}$$

When $n = 2$, ε is given as

$$\varepsilon = \frac{2Q}{3Q_S} + \frac{Q_S}{2Q} \tag{12}$$

The relation between Q_S and Q in Fig.2 (a) is as follows

$$Q_S/Q \geq 2 \tag{13}$$

The coefficient ε can be seen as a corrector of roughness in traditional assumption, therefore in the uniformly distributing assumption, the pipe has a different roughness, which is C_d in Hazen－Williams formulation,

$$\Delta H = \frac{10.67Q^{1.852}}{C^{1.852}D^{4.87}}L\varepsilon = \frac{10.67Q^{1.852}}{C_d^{1.852}D^{4.87}}L \tag{14}$$

and C_d is

$$C_d = \frac{C}{\varepsilon^{0.54}} \tag{15}$$

3. Pressure-dependent leakage

Water loss exists in every water distribution system, but the difference is in the amount. The most common classification of water loss is apparent loss and real loss. In addition, apparent loss is volumetric losses that can be represented as a function of the released water volume [5], while real loss is pressure-dependent. According to the awareness and severity, real loss can be divided into background leakage and pipe burst loss. The pipe burst formula is the classic orifice flow formulation [4]. The background leakage of one pipe can be expressed [4] as follows

$$q_{k-l} = \beta_k L_k \left(P_k\right)^{\alpha} \tag{16}$$

where P_k = average pressure in the pipe computed as the mean of the pressure values at both nodes of the kth pipe; and l_k = length of that pipe. Variables β and α = two leakage model parameters. α is 1.18 in this paper. β may be calibrated as follows

$$\beta = \frac{Q_T}{\sum_{i=1}^{N}\left(\sum_{k=1}^{M}\frac{L_k}{2}\left(P_k\right)^{\alpha}\right)} \tag{17}$$

where Q_T = system leakage, M = pipe number and N = node number. It is noted that the difference between water demand and leakage is that there is no fixed upper value for leakage [12]. And in this paper only the background leakage is investigated.

4. EPANET to perform hydraulic resistance correction model

The hydraulic resistance correction model is performed by getting an equivalent pipe roughness C_d. Due to the popularity, EPANET is used to simulate demand and leakage uniformly distributing along pipe. The leakage parameter β and C_d will be calibrated to carry out hydraulic resistance correction model by the following procedure:

(1) the q_d is firstly got from the water distribution system demand divided by total length of all the pipes discharging water. Then nodal demand is calculated by summing up half of pipe demands;

(2) a simulation is performed with EPANET to obtain β using Equation (17);
(3) nodal discharge is updated by adding nodal demand and leakage together;
(4) EPANET is run again to verify whether nodal pressure has the same value, if not, return step 2;
(5) comparing the system leakage in simulation and actual situation, if different, return step 2;
(6) β and nodal pressures P_S are acquired;
(7) pipe discharge is gained by summing pipe demand and pipe leakage;
(8) determining whether the pipe is fed by one node or both nodes by adopting Equations (10) and (13). And ε is gained using Equation (9) or (12) according to the distribution situations;
(9) the new roughness value is calculated by Equation (15) and assigned to the pipes;
(10) a simulation is performed with EPANET to evaluate whether nodal pressure satisfy convergence criterion, if yes, nodal pressures P_E are saved, if not, return step 6;
(11) the procedure is stopped when max $\{|P_i^S - P_i^E|/P_i^S \mid i \in [1, N]\} \leq 0.01$, if not, return step 2.

5. Case Study

Apulian network [4] is used as case study. The network has 23 nodes and 1 reservoir as shows in Fig. 3. The water demand in the system is 225.599L/s, system leakage rate is set to be 25% of system demand, which means the system leakage is 56.400L/s. It is noted that P-34 is a transmission main, which has no function of water distribution. The properties of nodes and pipes are given in Table 1, whose seventh column consists of the equivalent roughness C_d after considering hydraulic resistance correction coefficient ε. Fig.3 indicates that those pipes who have zero points flow inversion are fed by both ends, which means these pipes have far more discharge than pipe flow. The leakage of nodes and pipes are imported to ArcGIS, which shows the leakage clearly in Fig. 4 and Fig.5. As can be seen from the above two Figures, the leakage quantity of nodes and pipes are correlated to some extent with the distance from source. The nearer to source, the larger pressures they have, and the bigger leakage they hold. From Fig.3, and Fig.6 to Fig.7, with regarding to traditional distribution, the pressures of nodes close to zero points are overestimated, which lead to the leakage overestimation.

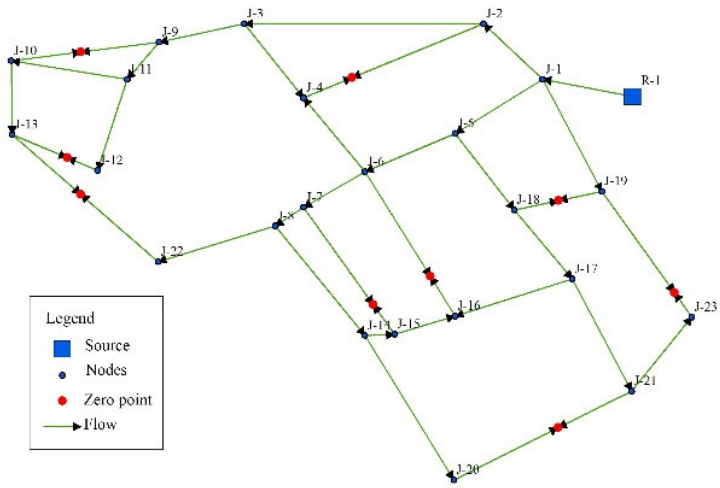

Fig.3. Layout and flow distribution of Apulian network

Table 1. Node and Pipe properties of Apulian network

Pipe ID	Start node	End node	L (m)	D (mm)	C (HW)	C_d (HW)	Node ID	$Q_{i\text{-design}}$ (L/s)	Elevation (m)
P-1	J-1	J-2	348.5	327	124	124.242	J-1	10.863	6.4
P-2	J-2	J-3	955.7	290	123	123.160	J-2	17.034	7
P-3	J-3	J-4	483	100	118	102.583	J-3	14.947	6
P-4	J-3	J-9	400.7	290	126	126.262	J-4	14.280	8.4
P-5	J-2	J-4	791.9	100	114	85.653	J-5	10.133	7.4
P-6	J-1	J-5	404.4	368	123	122.921	J-6	15.350	9
P-7	J-5	J-6	390.6	327	124	123.750	J-7	9.114	9.1
P-8	J-6	J-4	482.3	100	115	106.115	J-8	10.510	9.5
P-9	J-9	J-10	934.4	100	118	53.796	J-9	12.182	8.4
P-10	J-11	J-10	431.3	184	120	119.727	J-10	14.579	10.5
P-11	J-11	J-12	513.1	100	114	103.943	J-11	9.007	9.6
P-12	J-10	J-13	428.4	184	126	124.632	J-12	7.575	11.7
P-13	J-13	J-12	419	100	123	86.730	J-13	15.200	12.3
P-14	J-22	J-13	1023.1	100	119	54.738	J-14	13.550	10.6
P-15	J-8	J-22	455.1	164	121	119.388	J-15	9.226	10.1
P-16	J-7	J-8	182.6	290	125	125.181	J-16	11.200	9.5
P-17	J-6	J-7	221.3	290	123	123.202	J-17	11.469	10.2
P-18	J-1	J-19	583.9	164	118	116.647	J-18	10.818	9.6
P-19	J-5	J-18	452	229	122	121.573	J-19	14.675	9.1
P-20	J-6	J-16	794.7	100	115	81.976	J-20	13.318	13.9
P-21	J-7	J-15	717.7	100	116	83.881	J-21	14.631	11.1
P-22	J-8	J-14	655.6	258	127	126.524	J-22	12.012	11.4
P-23	J-14	J-15	165.5	100	112	111.855	J-23	10.326	10
P-24	J-15	J-16	252.1	100	124	111.572	R-1	N/A	36.4
P-25	J-17	J-16	331.5	100	116	110.435			
P-26	J-18	J-17	500	204	121	120.596			
P-27	J-17	J-21	579.9	164	120	118.812			
P-28	J-19	J-23	842.8	100	113	89.047			
P-29	J-20	J-21	792.6	100	121	60.402			
P-30	J-14	J-20	846.3	184	125	120.021			
P-31	J-9	J-11	164	258	126	126.082			
P-32	J-21	J-23	427.9	100	116	107.772			
P-33	J-19	J-18	379.2	100	138	55.176			
P-34	R-1	J-1	158.2	368	117	117.111			

While the pressures of those nodes without pipe flow inversion are underestimated, which result into lower leakage than actual situation. The reason is that discharge of pipe with flow inversion is much higher than pipe flow in simulation; the low pipe flow causes low pipe loss and high nodal pressure. While the situation of pipes without zero points are the opposite. This means pipes with flow inversion mainly distribute water to customers rather than transferring water, and these pipes have smaller diameters and higher hydraulic resistance correction coefficient, as can be seen from Table 1. This illustrates that distribution pipes should have some capacity surplus in WDSs optimization design.

6. Conclusions

This paper presents an improvement of traditional water distribution model to consider leakage and demand uniformly distributing along pipes, and hydraulic resistance correction procedure is performed by EPANET. The results presented in this article suggest that simulation of leakage control and pressure management should be represented as uniformly distributed rather than the traditional assumption does, especially when WDSs have pipes with flow inversion, the error of head loss is quite high, which cause overestimation of pressure and leakage, while the leakage of pipe without flow inversion is underestimated. And the pipes mainly distributing water to customers should have some capacity surplus to reduce head loss error in design.

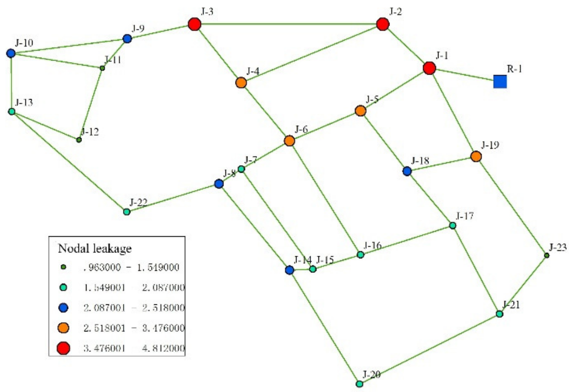

Fig.4. Node leakage of Apulian network

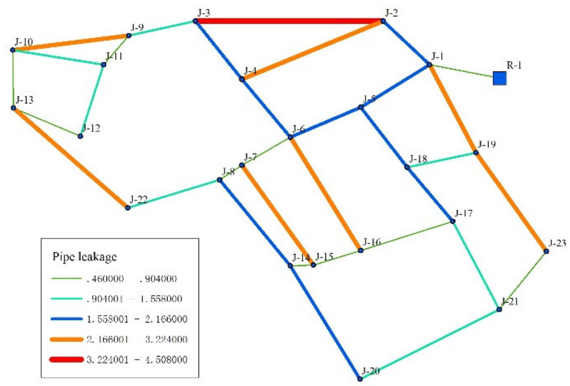

Fig.5. Pipe leakage of Apulian network

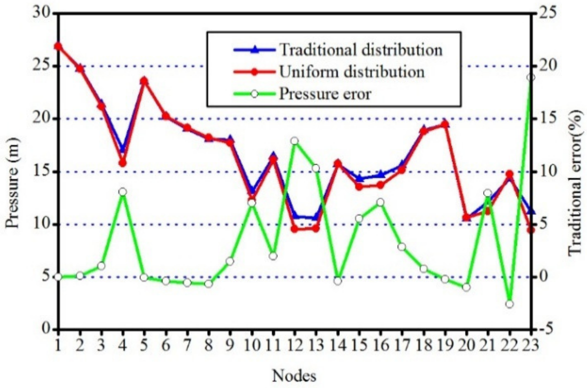

Fig.6. Comparison of Pressure resulting from different distribution assumptions

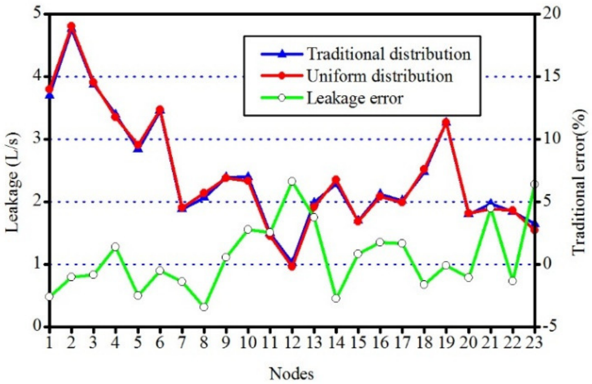

Fig.7. Comparison of Leakage resulting from different distribution assumptions

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